SUTTLE

Details of Railroad Girder Bridges

Civil Engineering

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DETAILS

OF

RAILROAD GIRDER BRIDGES

BY

CLIFFORD BRADLEY SUTTLE

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

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CLIFFORD BRADLEY SUTTLE

entitled DETAILS OF RAILROAD GIRDER BRIDGES

is approved by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

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I.

INTRODUCTION.

During the past ten years the use of plate girder bridges has become for spans up to 100 feet almost universal with leading railroads. On account of its simplicity of design, construction, and erection, the plate girder has displaced the trussed bridges which for the same spans contain less metal but are more complex.

Girders up to lengths which can be transported on three or four cars are now considered standard construction. Laws making necessary the elevation of tracks in large cities have tended to increase the use of plate girder bridges. Their durability and ease of erection and construction have been recognized to such an extent that the principal railroads have standard plans of plate girder bridges varying in length from 25 feet to 100 feet.

It is the purpose of this thesis to examine and discuss various features of standard designs for plate girder bridges.

II.

THE WEB PLATE.

1. Thickness.

The thickness of the web plate is never less than 3/8-inch. It has been found by inspectors that the web is the first to show signs of weakness, and for that reason it might be conducive to longevity if the minimum thickness were increased.

A peculiar feature of the Atchison, Topeka, and Santa Fe Railway's plans is that the web of all single track bridges is 3/8-inch thick, except in cases where head-room limits the depth. Their standards range from 26 feet to 105-1/2 feet. The Northern Pacific Railway uses 3/8-inch webs for lengths of 25 feet to 80 feet, and 7/16-inch from 80 feet to 100 feet.

When a double track is constructed on three girders, it is the usual practice to vary the thickness of the web of the center girder where ever the end shear calls for a plate thicker than that which can be punched. A saving of metal and the expense of reaming all holes in the web is thus effected. An example of variable web thickness is the 116-1/2 foot bridge of the New York Central and Hudson River Railroad. The web sections are 11/16-inch, 9/16-inch, and 7/16-inch thick. Table 1 shows the variations found in practice.

By building girders of such a length that a web

of variable thickness is necessary much of the simplicity and ease of transporting and erecting is lost. Such a girder cannot be assembled in a shop because the length is too great to admit of convenient transportation. It is highly probable that the economic length of girder is limited by the web to such as require a web which can be punched. A description of the longest span in America of this type of bridge will be given later.

2 Depth.

In standard designs there are considerable variations in the depth of the web plate. There is a difference in opinion as to the economic depth, but it is evident that the usual ratio of depth to length is 1/10 for spans up to 80 feet while as the length increases the ratio decreases. It may be as small as 1/12 for spans of 100 feet, and for still longer bridges it may reach 1/14. The Chicago, Milwaukee, and St Paul Railway constructed a single-track girder bridge at Janesville, Wisconsin, which had a span of 114-1/2 feet and a depth of 9-1/2 feet- a ratio of 1 to 12.

Ralph Modjeski's plans for the Northern Pacific Railway are good examples of modern plate girder design. His 100 foot deck girder has a depth of 8 feet. In the design of these plans Modjeski followed the practice of considering the web as taking part of the moment, and in girders which necessitated splicing of the webshe did not consider any part of them effective as flange area.

Henry Szlapka has deduced a set of formulas which were published in the Engineering Record. By the formu-

las, which the author states he has used in his practice, he endeavors to show that the economic depth varies from 1/7 to 1/9 of the span according to the amount of moment which the web is assumed to take. If his theory were strictly adhered to the economic length of girders would be 90 feet, which is obviously not correct. It is to be supposed that Szlapka's formulas are not practicable, because the assumptions used in deriving them are incorrect.

Johnson in his Modern Framed Structures, page 334, shows that the depth may vary 10% and cause a change in weight of only one half of one per cent. Practical considerations may be such as to make a slight increase in weight and decrease in depth preferable to theoretical economic depth with less weight. After examining a number of girders, a summary of which is given in the table following, we find a range of the ratio of depth to length, which varies from $\frac{1}{7.1}$ to $\frac{1}{13.7}$.

3. Stiffeners.

It has been shown by theory and experiments, that the forces acting in the web of a plate girder are at an angle of 45 degrees to the vertical, and that the tensile and compressive stresses intersect. In spite of the above facts, the practice of using vertical stiffeners is universal. If stiffeners were placed inclined the details of the girder would be such as to rob this style of construction of its main advantage- simplicity of erection. The fact that girders with vertical stiffening angles have

TABLE 1.

TABLE SHOWING VARIATIONS IN AREA OF WEB PLATE, AND RATIO OF DEPTH TO LENGTH.

Ref. No.	Owner	Length Over-all. feet.	D=Deck. T=Thru. s=single. d=double.	Depth of Web inches	Th'kness of Web inches	Area of Web sq. in.	$\frac{\text{Depth}}{\text{Length}}$
1	A.T. & S.F. Ry.	26	D-s	44	3/8	16.5	$\frac{1}{7.1}$
2	" "	30	T-s	47	"	17.6	$\frac{1}{7.7}$
3	M.St.P & S. Ste. M. Ry.	30 5/6	D-s	36	"	13.5	$\frac{1}{10.3}$
4	Belt Ry. of Chicago	39	T-d	41½	"	15.6	$\frac{1}{11.3}$
5	A.T. & S.F. Ry.	40	D-s	54½	"	20.4	$\frac{1}{8.8}$
6	" "	40	T-s	54½	"	20.4	$\frac{1}{8.8}$
7	Lake Erie & W. R	45 1/3	D-s	47¾	5/16	14.9	$\frac{1}{11.4}$
8	A.T. & S.F. Ry.	48	D-s	60½	3/8	22.7	$\frac{1}{9.5}$
9	" "	48	T-s	60½	3/8	22.7	$\frac{1}{9.5}$
10	T.C. R.R.	50	T-d	72	1/2	36.0	$\frac{1}{8.3}$
11	N.P. Ry.	60	D-s	71¾	3/8	26.9	$\frac{1}{10.}$
12	A.T. & S.F. Ry.	64	D-s	73½	3/8	27.6	$\frac{1}{10.5}$
13	" "	64	T-s	73½	"	27.6	$\frac{1}{10.5}$
14	C.M. & St. P.	65	T-s	73	7/16	31.9	$\frac{1}{10.7}$
15	Belt Ry. of Chic.	66	T-d (c.g)	83½	1/2	41.8	$\frac{1}{9.5}$
16	" " " "	66	T-d (o.g)	83½	3/8	31.3	$\frac{1}{9.5}$
17	C & A. Ry.	70	D-s	92⅜	7/16	40.4	$\frac{1}{9.1}$
18	A.T. & S.F. Ry.	75	D-s	84½	3/8	31.7	$\frac{1}{10.7}$
19	C.M. & St. P. I	75	D-s	99½	7/16	43.5	$\frac{1}{9.1}$
20	P.C.C. & St. L. Ry.	74 1/6	D-s	83½	3/8	31.3	$\frac{1}{10.7}$
21	U.P.R.R.	80	T-s	96	7/16	42.0	$\frac{1}{10.0}$
22	N.P. Ry.	80	D-s	83¾	7/16	36.6	$\frac{1}{11.4}$
23	U.P.R.R.	80	D-s	96	7/16	42.0	$\frac{1}{10.0}$
24	C.B. & Q	86	T-s	96	9/16	53.9	$\frac{1}{10.8}$
25	A.T. & S.F. Ry	90	D-s	99½	3/8	37.3	$\frac{1}{10.9}$
26	C.C.C. & St. L. Ry.	91½	D-s	90	3/8	33.8	$\frac{1}{12.2}$
27	U.P.R.R.	95	T-s	108	1/2	54.0	$\frac{1}{10.5}$
28	A.T. & S.F. Ry.	100	D-s	108	3/8	40.5	$\frac{1}{11.1}$
29	N.P. Ry.	100	D-s	95¾	7/16	41.9	$\frac{1}{12.7}$
30	" "	100	T-s	101¾	7/16	44.5	$\frac{1}{11.8}$
31	A.T. & S.F. Ry	105½	T-s	113½	3/8	42.6	$\frac{1}{12.2}$
32	C.B. & Q. Ry.	105	T-s	120	5/8	75.0	$\frac{1}{10.5}$
33	Ind. H. R.R.	110	T-s	120	7/16	52.5	$\frac{1}{11.0}$
34	Chic. Term. T. Co.	115½	T-s	119	5/8	74.4	$\frac{1}{11.6}$
35	Erie R.R.	131⅓	D-s	113¾	1/2	56.9	$\frac{1}{13.7}$

been used extensively, and found to be satisfactory, is sufficient proof that this practice is on the side of safety.

The usual spacing of stiffeners is that specified by Cooper - "not more than five feet apart,nor a greater distance than the depth of girder." The end angles should not be placed more than 18 inches apart. In some designs the spacing of stiffeners is varied from 18 inches at the end, to the maximum of 5 feet at the center. The end stiffeners are designed to resist the maximum shear, while the intermediate angles are made of equal size and usually are not proportioned to resist the shear at the points at which they are placed. Probably the most used shapes for intermediate stiffeners are the 5" x 3-1/2" x 3/8" angles. They are placed with the 5-inch leg out-standing, and their use for spans of 75 feet to 100 feet, spaced according to Cooper's Specifications, might be taken as a rule of thumb for designing.

Frank W. Skinner in the Engineering Record of Oct. 7, 1905, gives a number of condensed specifications for stiffeners as required by different railroads and bridge companies. He says that there are no technical reasons for such great variations in specifications of different roads for structures for the same service. It appears that there is a great diversity of opinion among engineers as to the spacing of stiffeners, and their action under stress.

In examining the specifications given in Skinner's

article a great many details of importance are seen to be lacking. Some do not mention whether stiffeners are to be crimped or fillers used, while others do not give any specifications except for spacing. Again, in some the depth of girder governs the size of stiffeners, while the Southern Railway Company vary them according to the ratio of the depth of web to its thickness. The Lake Shore and Michigan Southern Railroad specifies that certain angles shall be used on girders of certain length. The Lehigh Valley Railroad specifies that for girders 3-feet high and under, angles not less than 3" x 3" x 3/8" shall be used; for girders 10 feet high not less than 6" x 3-1/2" x 3/8"; for girders between 3 feet and 10 feet high stiffeners shall be proportionate to the sizes stated.

Assuming a girder 75-feet long with a depth of 7 feet and a thickness of web of 3/8-inch, the size of stiffener angles may be 4" x 3" x 5/16", 4" x 3" x 3/8", or 5" x 3-1/2" x 3/8", according to different specifications. If we assume the unsupported distance between flange angles to be 75 inches, we can get a range of maximum spacing varying from 5 feet to 14 feet as shown in Table 2.

In view of the fact that so little is known concerning the function of the stiffeners, the specifications given by Cooper in regard to spacing should be accepted as this gives the greatest safety. There is a difference of opinion as to when crimping, and when fillers should be used on intermediate stiffeners. It is almost universally

TABLE. 2. VARIATIONS IN SPECIFICATIONS FOR INTERMEDIATE STIFFENERS

Assumed Length of Girder 75 feet.
" " Thickness of Web 3/8 inch.
" " Depth " " 7 feet.
" " Unsupported Depth 75 inches.

Ref. No.	Specifications	Size of Stiffener	Maximum Spacing	Date	Remarks
1	American Bridge Co.	Not Spec.	5 feet.	1900	Crimped
2	A.T. & S.F. Ry.	" "	$10\frac{1}{2}$ "	1902	
3	B. & O. R.R.	5" x $3\frac{1}{2}$" x 3/8"	6 "	1904	
4	Choct. Ok. & Gulf Ry.	Not Spec.	14 "	1902	fillers
5	Canadian Pac. Ry.		7 "	1903	
6	C.C.C. & St. L. Ry.	4" x 3" x 3/8"	$5\frac{1}{2}$ "	1901	Crimped
7	Coopers Spec.		5 "	1901	
8	Erie R.R.	Not Spec.	6 "	1900	
9	King Bridge Co.	4" x 3" x 5/16"	7 "	1898	Crimped
10	Lake S. & M.S. Ry.	5" x $3\frac{1}{2}$" x 3/8"	5 "	1904	
11	Mich. Cent. R.R.	Not Spec.	8 "	1904	
12	N.Y. Cent. & H.R.R.R.		6 "	1904	fillers
13	Osborn Eng. Co.		6 "	1903	
14	Phoenix Bridge Co.	Not Spec.	7 "	1895	
15	Phil. & Reading Ry.	4" x 3" x 3/8"	7 "		
16	Penn. Lines, W. of Pitts.	Not Spec.	7 "	1897	fillers
17	Southern Ry.	5" x $3\frac{1}{2}$" x 3/8"	5 "	1902	fillers
18	Union Pac. R.R.	Not Spec.	7 "	1904	
19	Lehigh Valley R.R.	5" x $3\frac{1}{2}$" x 3/8"	5 "	1899	

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specified that end stiffeners shall have fillers. In some instances the end of the plate is stiffened,not only by means of angles, but also with two reinforcing plates. All angles are placed in pairs, one on each side of the web, and are ground to fit very closely up to the flange angle.

4. Splices.

There are a variety of styles of web splices in standard girder design. The length of plate rolled for any required depth determines in a way the position of splices, but this length is usually decreased in order that splices may be placed at stiffeners.

Six methods of splicing the web are shown on Plate 1. Fig. 1 shows the simplest form, and is capable of effectively resisting shear only. An objection to this form is that the splices are proportioned for shear only, and the number of rivets in such splices is often so small as to render it certain that those near the top and bottom are over strained by bending.

In the other types shown a plate is placed at the top and bottom of the splice, and enough rivets are put in to resist the bending stresses on the section. The bending stresses being zero at the neutral axis, and increasing uniformly toward the flanges, it is necessary that the rivets near the top and bottom be space closer than those near the center. In order to provide sufficient space for the rivets where the bending stresses are a maximum, plates

PLATE 1.
TYPES OF STANDARD WEB SPLICES.

Fig. 1.

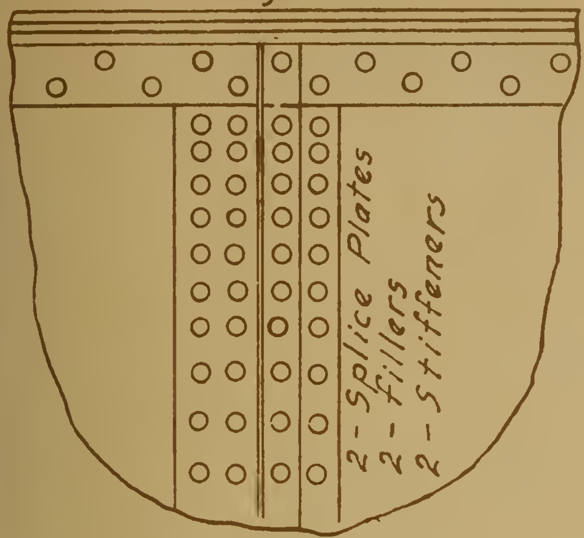


Fig. 2.

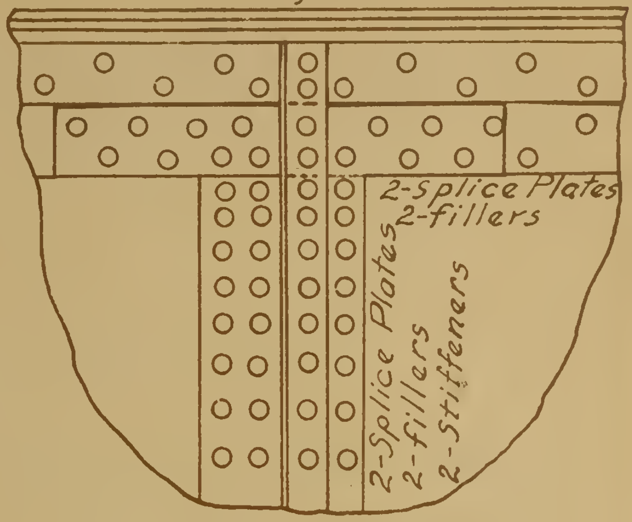


Fig. 3.

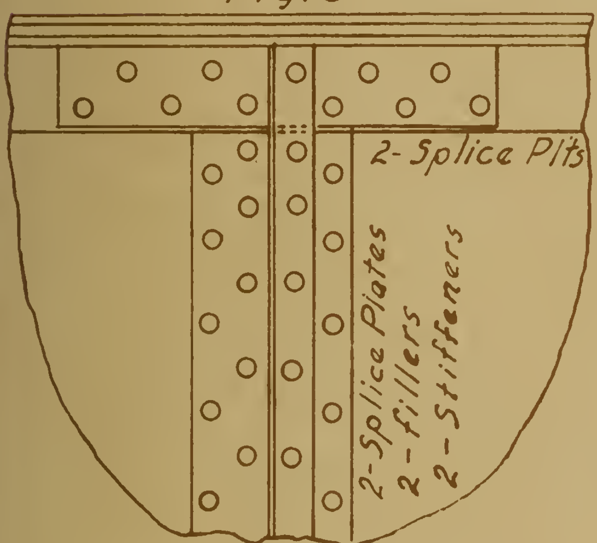


Fig. 4.

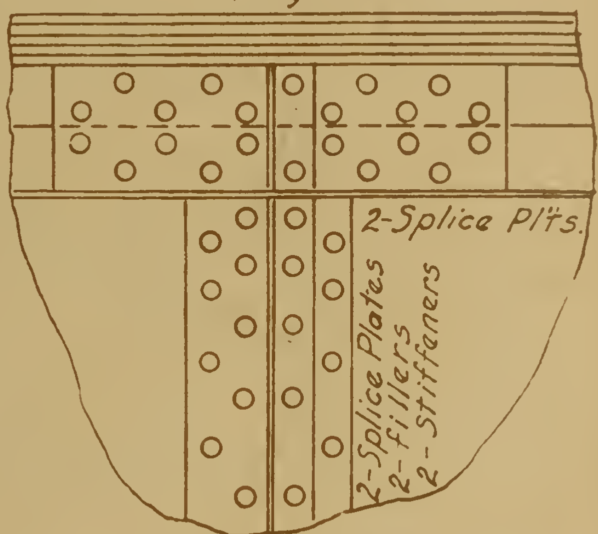


Fig. 5.

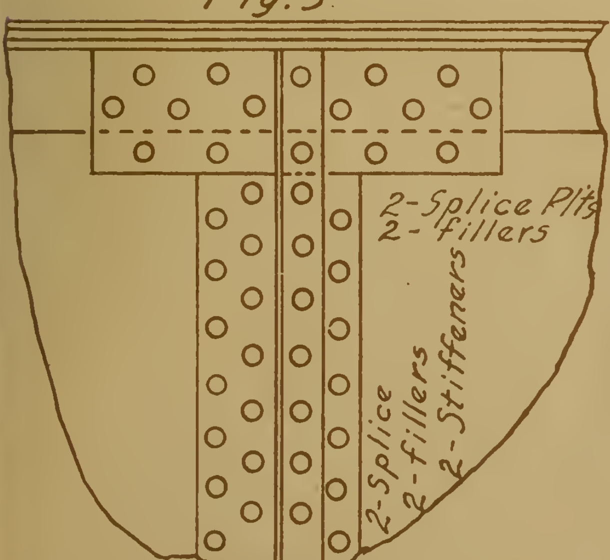
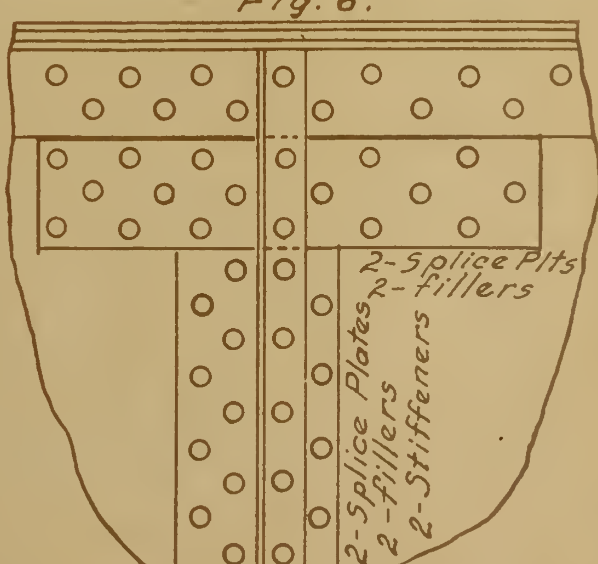


Fig. 6.



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with their length parallel to the length of girder, are used. The spacing of plates and rivets are shown in Plate 1.

III.

THE FLANGE.

1. Forms.

The most common flange is that shown in Fig. 1, Plate 2, and consists of two angles and one or more cover plates. It is used on all girders in which one half of the chord stress does not require angles larger than 8"x8". It will be found that 6" x 6" angles are usually used on spans up to 80 feet, and the 6" x 8" and 8" x 8" angles for longer spans. A feature of note is that, with one exception, all flange angles of the Atchison, Topeka and Santa Fe Railway standards are 6" x 6". That exception is the Class A, 100-foot deck girder, upon which 8" x 8" x 7/8" angles are used.

Frequently in order to increase the bearing value of the rivets so that it will be equal to, or greater than, their value in double shear, a plate will be introduced between the web and angles as shown by Fig. 2. The plates may be extended to engage more than one row of rivets.

For very heavy girders a flange, Fig. 3, consisting of four angles and several cover plates, is used extensively. Side plates are sometimes put into this form of flange.

PLATE 2.
TYPES OF STANDARD FLANGES.

Fig. 1.

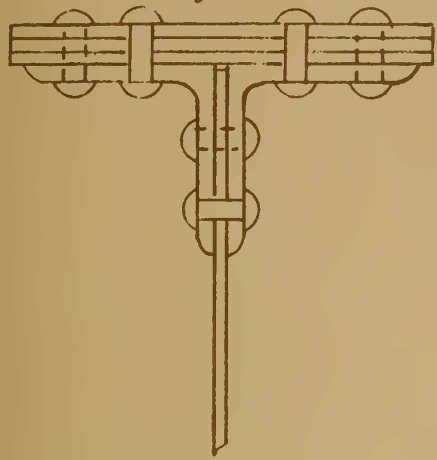


Fig. 2.

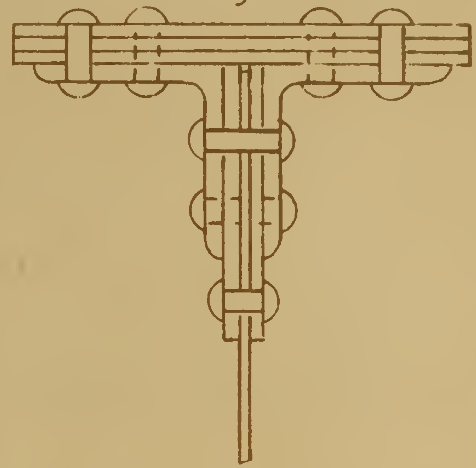


Fig. 3.

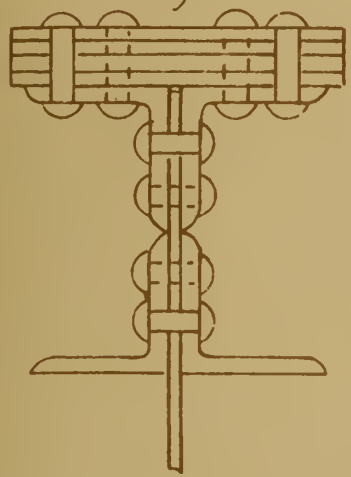


Fig. 4.

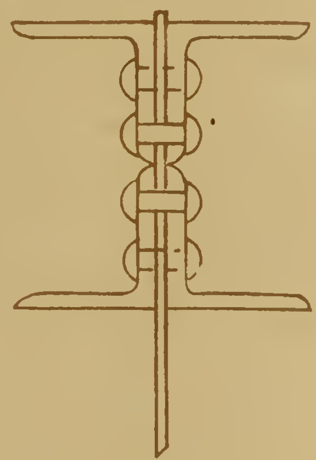


Fig. 5.

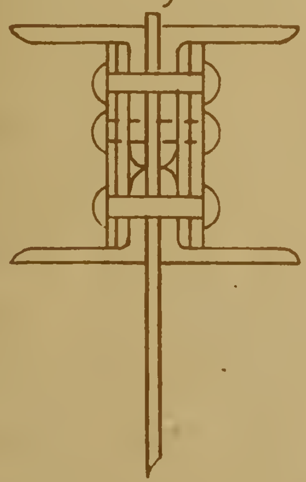
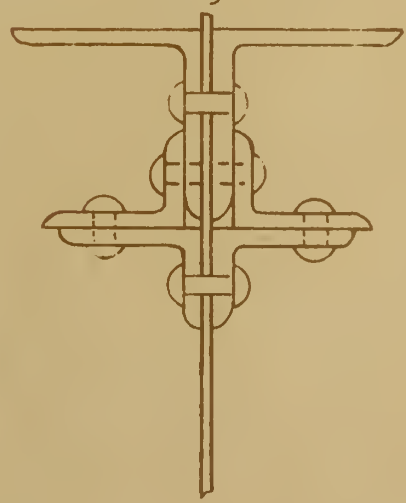


Fig. 6.



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Types shown by Fig. 1, 2 and 3, are the most common, but lately there has come into use for deck girders three forms of top flanges which are completely of angles. The bottom flanges are made up in the usual manner. Fig. 4, 5, and 6, Plate 2, show the arrangement of the top flanges. The lower angles do not extend the full length of the girders, the lower one of Fig. 6 being shorter than the middle angle, which in turn is less than the length of the girder. The top angle is full length.

The angle form of top flange brings into its composition members which are best suited to resist compression. It forms a smooth bearing for the ties which are notched over the 3/4-inch projection of the web plate. It is here that the weakness lies. By this construction the angles receive the direct action of wheel loads, and it is a fair supposition that they will work loose. The flange plates of Fig. 1, 2, and 3 not only take up stress but they protect the angles and give stiffness; remove the plates and the angles will evidently be under a disadvantage. The cracks between the web and the angles will form convenient receptacles for quantities of water, which are not to be desired because it will reach the rivets in addition to weakening the flange angles by rusting them. The effective depth of both girder and web is decreased by the angle top flange. The last is not a very serious defect because a girder may vary 10% from the effective depth, and increase the weight only 1/2 of one per cent.

It is quite common practice to fasten the lateral bracing to the bottom angle. The efforts of wind and lateral vibration are applied at the top of the flange and acting on a lever with the stiffeners as a point of support, tend to tear the lower angle away from the web.

The Chicago and Alton Railway use the angle flange shown in Fig. 5, and the Chicago, Milwaukee and St. Paul Railway use the type of Fig. 6, for standard deck girders. The latter attempts to overcome the difficulty stated above. Short lengths of stiffener angles are placed between the outstanding legs of the flange angles, and the lateral bracing is connected to the top angle. The connection plate for the cross framing is cut out to clear the lower angle. The detail is objectionable because it allows a limited space to work in and considerable shop expense is connected with its use.

Channels are not adaptable for flanges of plate girders. The flanges of the channels are too narrow to permit the use of cover plates of sufficient width to give the necessary stiffness, and the webs are usually too thin to allow stable connections without the use of an excessive number of rivets.

The specifications given by Cooper state that one half of the total flange area must be angles. This is not rigidly followed. In Table 3, the per cent of total flange area which is composed of angles, varies from 24 to 69. There is no specific law governing the variation, but in

TABLE 3.

Table showing Variations in Flange Areas and Ratio of Area of Angles to Total Flange Area. Single Track Bridges

Ref No.	Owner	Length o to o. FEET	D=Deck T=Thru	Angles in Flange	Plates in Flange	Area Plates + Angles	Aang's / Apls+An's
1	A.T & S Fe Ry	26	D-S	2-6×6×7/16	1-14×7/16	16.24	.62
2	" "	30	T-S	2-6×6×1/2	1-14×3/8	16.75	.69
3	M.St P & S St M Ry.	30'-10"	D-S	2-6×6×5/8	1-12½×5/8	22.03	.65
4	A T & S Fe Ry.	32	D-S	2-6×6×½	1-14×7/16	17.02	.65
5	" "	40	D-S	2-6×6×½	2-14×½	25.5	.45
6	" "	40	T-S	2-6×6×9/16	1-14×½	25.98	.48
					1-14×7/16		
7	L.E. & W. R.R.	45⅓	D-S	2-5×3½×9/16	3-12×3/8	22.44	.40
8	A.T. & S Fe Ry.	48	D-S	2-6×6×¾	1-14×5/8	29.97	.48
					1-14×½		
9	" "	48	T-S	2-6×6×¾	2-14×½	30.88	.48
10	" "	50	D-S	2-6×6×⅞	2-14×¾	40.48	.40
11	I.C R R.	50	T	2-6×6×¾	2-14×¾	42.38	.40
					1-12×3/8 S.P.		
12	N.P. Ry.	60	D-S	2-6×6×5/8	1-12½×½	31.43	.45
					2-12½×7/16		
13	N.P. Ry.	60	T-S	2-6×6×¾	3-14×3/8	32.58	.52
14	A T. & S. Fe Ry.	64	D-S	2-6×6×⅞	3-14×½	40.48	.48
15	" "	64	T-S	2-6×6×⅞	3-14×½	40.48	.48
16	C & A. R R	70	D-S	2-6×6×¾	2-15½×11/16	47.98	.35
					1-15½×5/8		
17	C.C.C & St.L Ry.	74⅙	D-S	2-6×6×5/8	3-14×½	35.22	.40
18	C.M & St.P Ry.	75	T-S	2-6×6×¾	1-16×¾	47.98	.35
					2-16×5/8		
19	A.T. & S Fe Ry.	75	D-S	2-6×6×⅞	2-14×5/8	48.48	.40
					1-14×¾		
20	" "	75	T-S	2-6×6×⅞	3-14×5/8	45.58	.45
21	U. P. Ry.	80	T-S	2-8×8×¾	4-18×5/8	67.88	.33

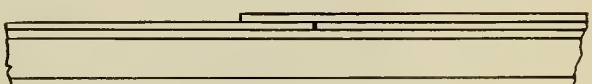
TABLE 3.—contd.

Ref No.	Owner	Length o. to o. FEET	D=Deck T=Thru	Angles in Flange	Plates in Flange	AREA Plates + Angles	$\frac{A_{ang's}}{A_{pls}+A_{n's}}$
22	A.T. & S. Fe Ry	80	D-s	2-6×6×7/8	2-14 × 1/2	42.23	.46
					1-14 × 5/8		
23	U. P. R.R.	80	D-s	2-6×6×3/4	3-16 × 3/4	52.88	.32
24	N. P. R.R.	80					
25	A T & S Fe Ry	90	D-s	2-6×6×7/8	3-16 × 1/2	55.48	.35
					2-12 × 1/2 s.p.		
26	" "	90	T-s	2-6×6×7/8	4-16 × 5/8	59.5	.33
27	C.C.C. & St L Ry	91 1/2	D-s	2-6×6×7/8	4-16 × 9/16	55.5	.35
28	N.P. Ry.	100	T-s	2-6×8×3/4	3-18 × 9/16	68.3	.34
					2-18 × 1/2		
29	A.T. & S. Fe Ry	100	D-s	2-8×8×7/8	1-18 × 9/16	61.3	.43
					1-18 × 5/8		
					1-18 × 3/4		
30	" "	100	T-s	2-6×6×7/8	2-20 × 9/16	66.5	.29
					1-20 × 5/8		
					2-12 × 1/2 s.p.		
31	N.Y., N.H. & H. R.R.	103		2-8×8×3/4	4-21 × 5/8	106.6	.37
				2-6×6×3/4	2-14 × 1/2 s.p.		
32	C. B. & Q. R.R.	105		2-8×8×3/4	3-19 × 11/16	80.	.33
					1-19 × 3/4		
33	A.T. & S Fe Ry	105 1/2	T-s	2-6×6×7/8	3-20 × 5/8	70.	.28
					2-12 × 1/2 s.p.		
34	Ind. H. R.R.	110		2-8×8×7/8	2-20 × 3/4	109.	.24
					3-20 × 5/8		
					2-15 × 1/2 s.p.		
35	Erie R.R.	131 1/3	D-s	4-6×8×7/8	4-20 × 5/8	95.92	.48

general the per cent of angles decreases as the length of girder increases. The per cents given in the table would be much smaller if 1/8 of the area of the web had been considered effective flange area. As given they are sufficient for comparison.

2. Splices.

Flange splices are made with plates or angles, or with both. The angle splices are staggered and made symmetrical with respect to the center of the span. Fig. 1 and 2, Plate 3, illustrate the method of splicing the angle. The splice angles are sheared to fit inside the flange angle. In some designs the splice angle on the opposite side of the web is omitted or simply a plate used on the vertical leg. In Fig. 2 it is seen that three plates, one on each side of the web and one on the outstanding leg of the angle spliced, form the splice. The cover plates are spliced by lapping the plates.



Cover plate Splice

IV.

LATERAL BRACING.

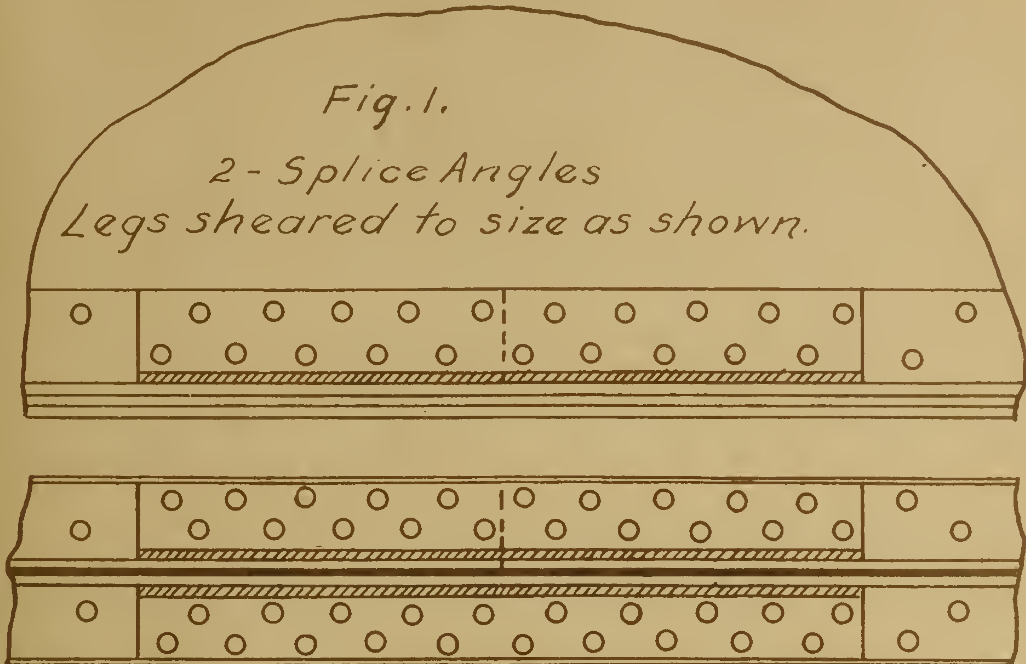
1. Forms.

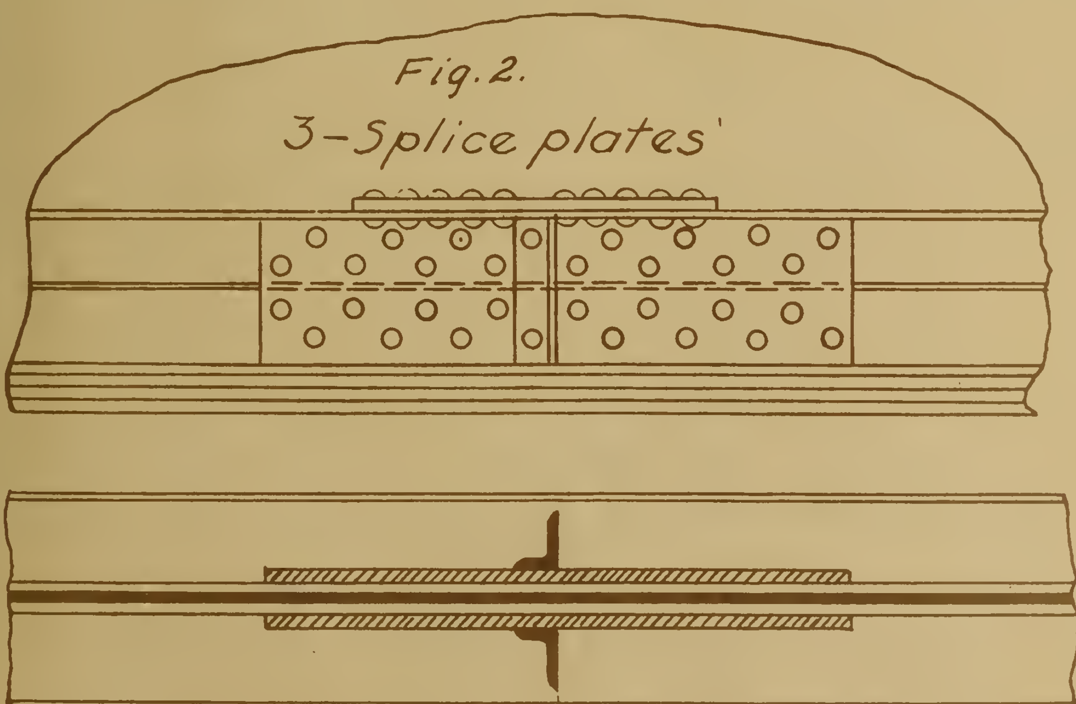
There are three styles of bracing in standard practice. The Warren type with sub-struts is the most prevalent. The diagonal members are designed to resist reversed stresses, and to give a maximum stiffness with a minimum amount of metal.

The diagonals of the cross-brace , the other form

PLATE 3

TYPES OF STANDARD FLANGE ANGLE SPLICES





C.B Suttle

in common use, are designed for tension only, and is the bracing used by those who do not believe in subjecting a member to alternating stresses. On long girders which require large shapes for the bracing, the cross-brace is the favorite type. This style was used on the 131-foot girder bridge of the Erie Railroad; the diagonal members being each composed of two angles. The cross-struts are usually placed at stiffeners.

The simple Warren bracing is sometimes used on short spans, but is not stiff enough for long bridges. The cross frames of deck girders are of the cross-bracing style. The end frames are made heavy enough to carry one half of the total wind load. The selection and placing of intermediate frames is a matter of practice.

Some designs place a filler between the underside of the flange angle and the connecting plate of the bracing, in order that the rivets in the plate may not come into contact with the tie which is notched over the flange. This is a useless detail whenever the notch in the tie is not deeper than a single cover plate, and the thickness of the flange angle is greater than the depth of rivet head. When there is a succession of girder spans, the lateral bracing is sometimes made continuous throughout the length. The girders may be fastened together by rivets through the end stiffeners, a filler being placed between them. This arrangment serves to unite the several spans, and relieves the pier masonry by carrying the wind reactions on to the abutments.

2. Shapes used in construction.

On some of the first girders constructed the laterals were all tension members, and composed of round rods. Round members are never used in modern designs on account of the rapid decrease in strength caused by the action of the elements, and the great vibration allowed by them. Angles are now almost universally used, although in exceptional cases plates have taken their place. The Illinois Central Railway used plates for laterals on some of their sub-way girders. For the end cross-frames of heavy girders, the Northern Pacific Railway designs require seven-inch channels. The angles used in the laterals are often fastened by both legs, in order to develop the full strength of the member. The practice of using 3-1/2" x 3-1/2" x 3/8" angles for intermediate cross-frames spaced every two or three panels, is quite common for all spans.

The advantages of plates for tension members of lateral bracing are becoming recognized. There is no waste of metal and in addition to being economical, they are easy to handle. The only disadvantage is a tendency to sag; but on through bridges sagging can be prevented by attaching the plates to the stringers.

V.

FLOOR SYSTEMS.

1. Deck bridges.

On deck girders the ties are notched over the flanges to form the floor system. Every fourth tie is fast-

PLATE 4.

TYPES OF STANDARD FLOORS DECK GIRDER BRIDGES

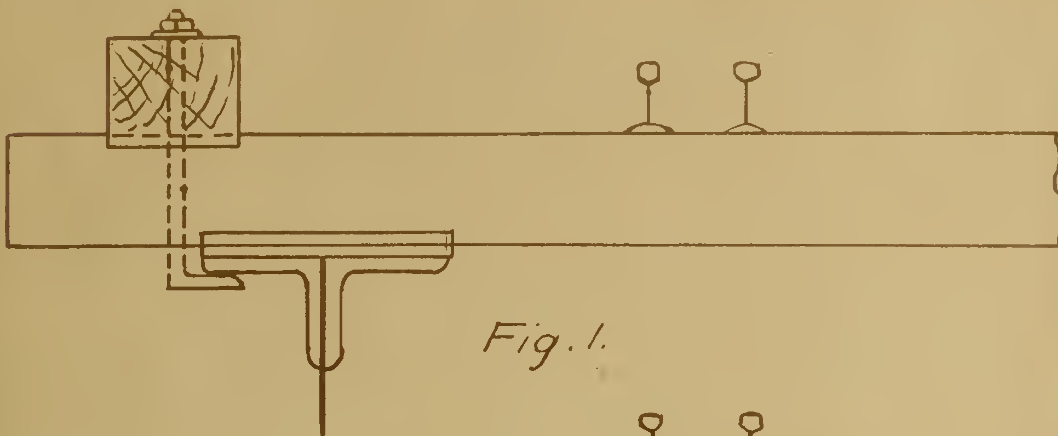


Fig. 1.

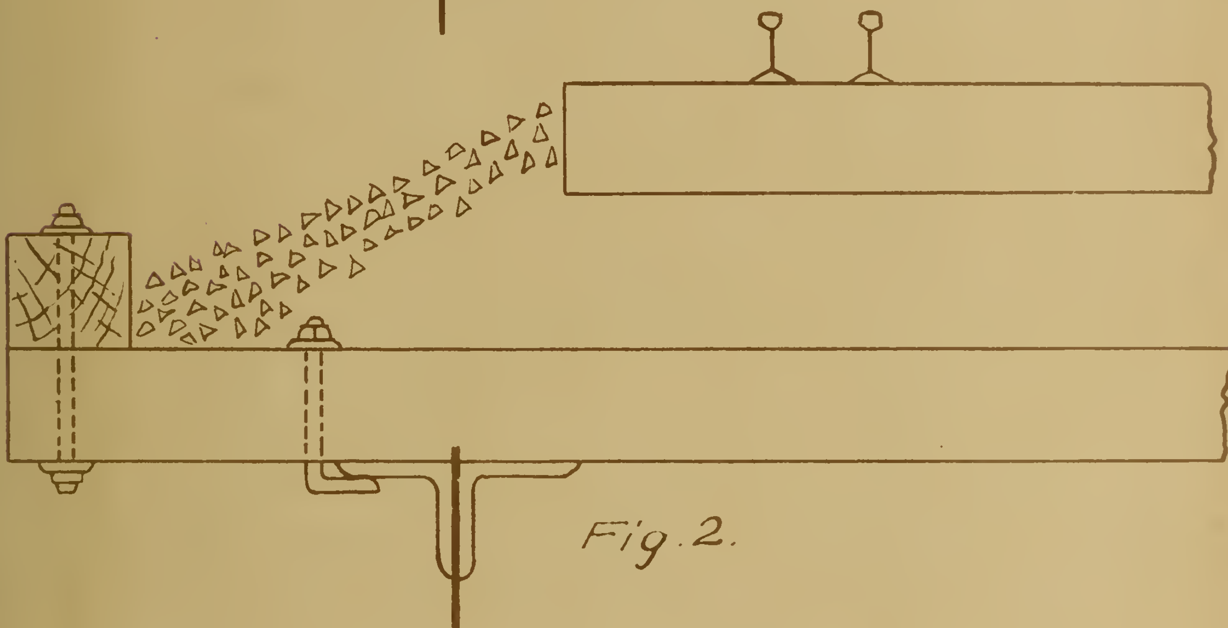


Fig. 2.

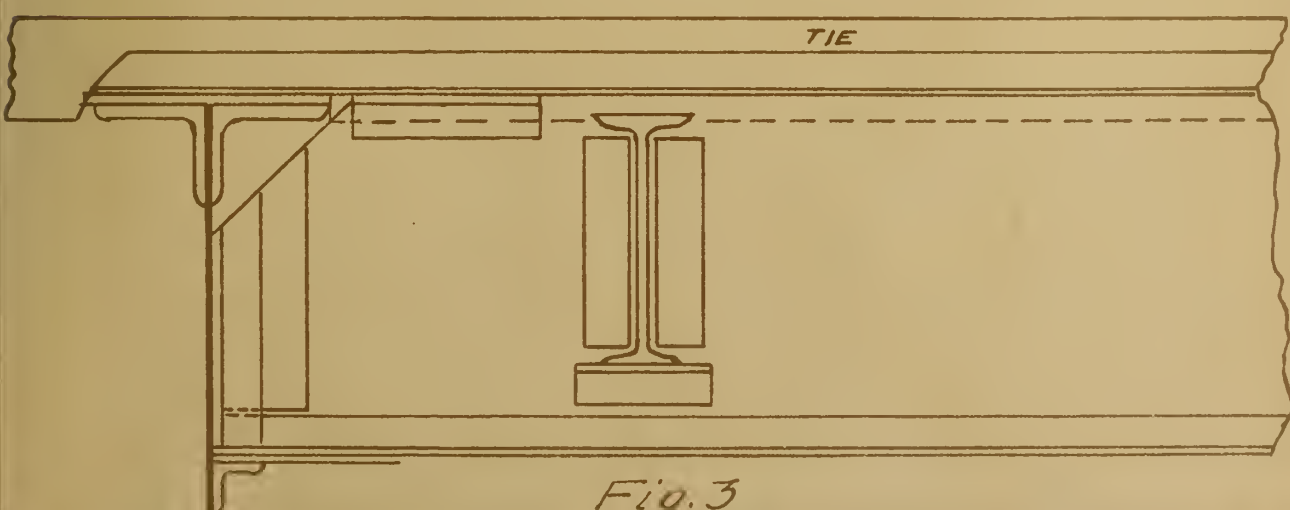


Fig. 3

ened to the flange by a bolt. The details are shown in the figures in Plate 4.

A solid floor is made by placing the ties as near together as possible and putting ballast on them. The ballast retards the drainage of water and becomes saturated, consequently rotting the ties.

Stringers are not generally used in the floor systems of deck plate girder bridges. The standard floor for deck girders on the Boston and Maine Railroad, Fig. 3, Plate 4, has stringers placed five feet on centers, with their tops flush with the tops of the girders. Four supports for the ties are thus available, and the top flanges are relieved of the direct action of wheel loads. The system constitutes a large part of the bracing, and the cost is greater than that of the ordinary floor by an amount equal, approximately, to two suits of ties. The system is to be commended for very deep girders, but the practice of placing deck girders 6 1/2 or 7 feet on centers is doing away with the need for stringers.

2. Through bridges.

The floor systems for through girder bridges, Plate 5, consist essentially of cross floor beams connecting the girders and stringers between the beams. The ties are notched over the stringers. Where a shallow floor is desired the ties are set on angles riveted to the webs, or directly on the bottom flanges, Fig. 3, Plate 5. The ties used must be long, and the deflection of them makes the

PLATE 5. THROUGH GIRDER BRIDGES
TYPES OF STANDARD FLOOR SYSTEMS.

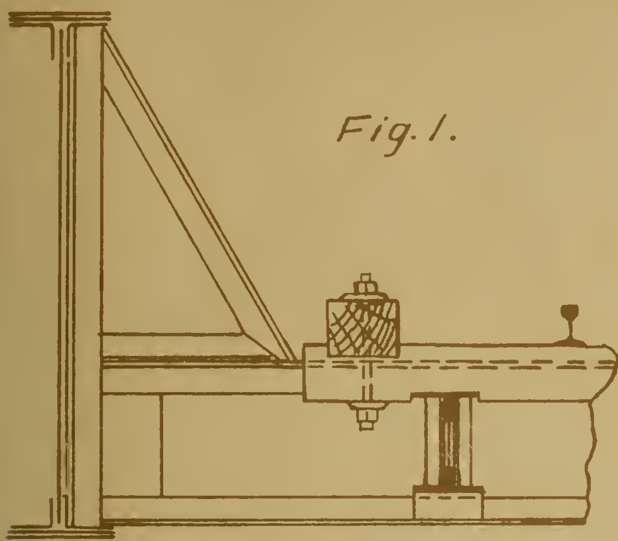


Fig. 1.

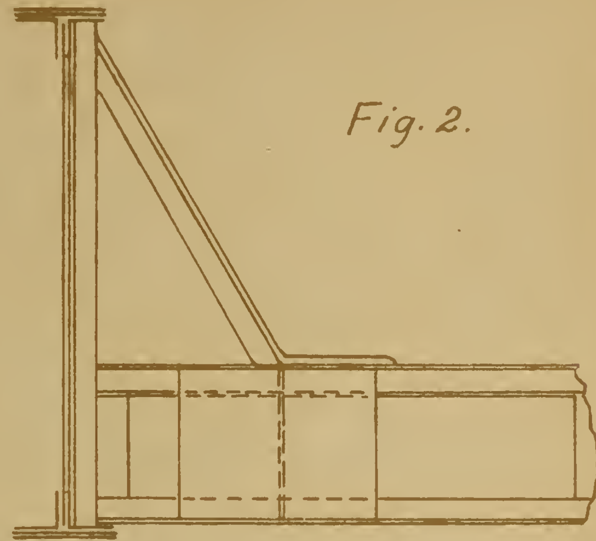


Fig. 2.

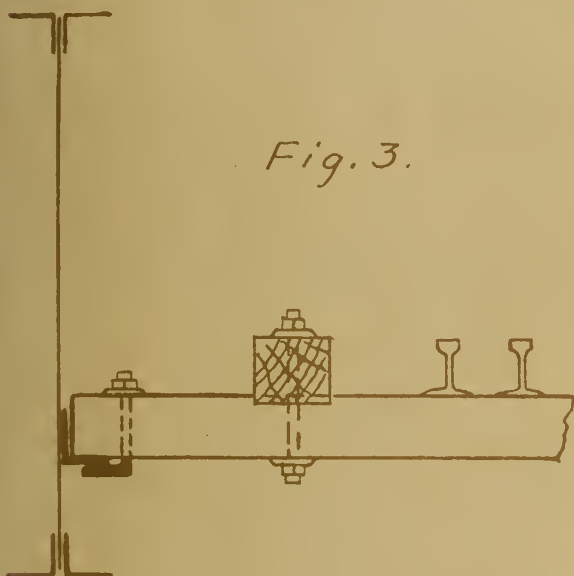


Fig. 3.

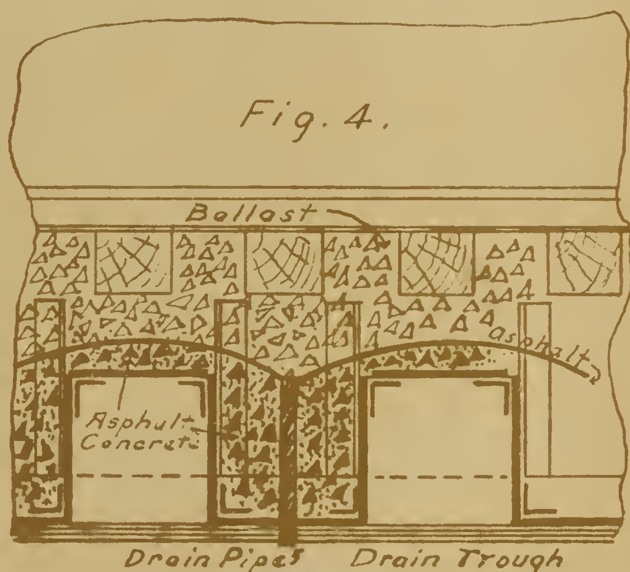


Fig. 4.

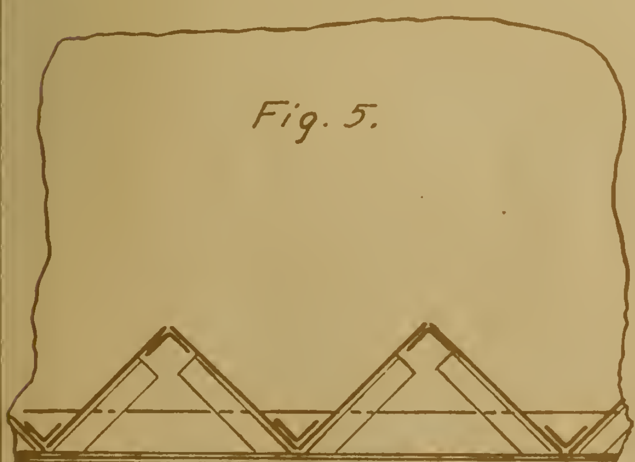


Fig. 5.

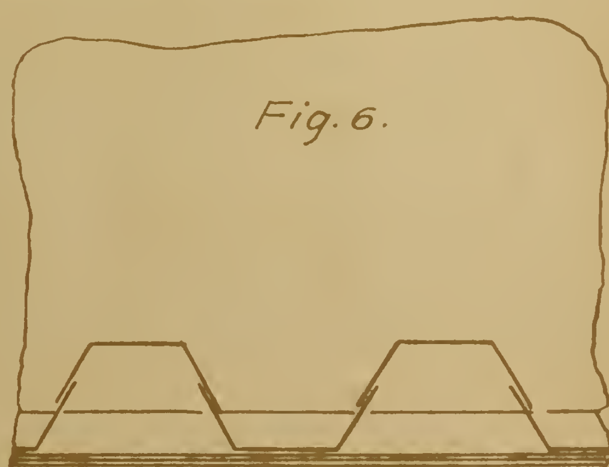


Fig. 6.

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floor unreliable. When the supporting angles are used the web is very likely to be distorted and unduly stressed. It is never proper to transfer stress to one side of the flange.

The best and most elaborate floor system consists of gusset plates, attached to the web, to which the floor beams are connected. The stringers are placed between the cross-beams, about 5 feet on centers. The gusset plates take the place of top laterals and prevent lateral motion of the top flanges. See Fig. 1 and 2, Plate 5.

In old designs the gusset plate reached only to the top of the cross-beam, to which it was fixed by a pair of angles. The cross-beam extended from one web plate to the other. The gusset plate was attached to the web plate with two angles, Fig. 1, Plate 5.

A better design than the one described above, continues the gusset plate down to the bottom of the floor beam, to which it is attached by splice plates, Fig. 2, Plate 5. The point of the reaction is thus moved toward the center of the bridge, the moment is decreased, and the gusset plate transfers the stress to the web plate. It is common practice to stiffen the edge of the gusset plate with a pair of angles. Unless bent over and riveted to the under side of the upper flange angles, the stiffeners are not of much value as strengtheners. The floor beams and stringers are made of built up or rolled sections. Rolled beams are used when the panel length is short.

Some designs rest those stringers in the end panels

on the masonry. The best designs provide an end floor beam. The advantages of the end floor beam as stated by J. A. L. Waddell, are as follows; "It allows a homogeneous motion of contraction and expansion for all the metal in the structure, with only two places for sliding or rolling motion, a greater rigidity of the floor system in the end panels, and a very satisfactory end lower lateral strut."

When traffic is to pass under a bridge there must be protection from falling coal, ashes, water, and other things which might occasion damage or discomfort. In residence districts it is desirable to subdue the rumbling caused by passing trains. The solid floor is an attempt to fulfill these necessary requirements for city use. It is possible to prevent the greater nuisance, but trains passing on girder bridges will cause more or less disturbance by creating a dull, disagreeable noise.

The types of solid floors now in use are made of rolled shapes and plates. The floor shown by Fig. 4, Plate 5, made up of plates and angles is the most widely used. Fig. 5 and 6 are two other forms of solid floor. The solid floors weigh from 30 pounds to 40 pounds per square foot. In his formulas for steel weights of through girder bridges, Johnson allows 300 pounds per foot, extra, for solid floors. These floors are rigid, durable, and watertight, but are costly. The troughs are usually filled with concrete, either cement or asphaltic. The ties are placed on ballast as on a roadbed.

VI.

BEARINGS.

Almost all railroads have standard bearings for different lengths of girders. There are two general types of bearings; cast bearings, both cast-iron and cast-steel; and bearings made up of plates and shapes.

Cooper"s Specifications are generally adhered to in the design of bearings. Bridges less than eighty feet in length are secured to the masonry at one end, and the other is free to move upon a smooth surface; a finished plate being fastened to the masonry and one to the girder. Rollers may be used. All bridges over eighty feet in length have hinged bolsters under both ends and a form of expansion bearing at one end. The expansion may be taken up by the movement of rollers moving between two plates, or by the tipping of segments. On the Nothern Pacific Railway all spans under sixty-five feet are without rollers. The Chicago, Milwaukee and St Paul Railway does not use rollers on spans of less than ninety feet.

There has been some discussion among engineers as to the relative merits of cast iron and cast steel when used in bearings. Those who favor cast iron claim that it will stand twenty five years of service without appreciable deterioration, and that wrought iron and cast steel often pit so as to become useless after five years of service. The cost of cast steel is twice that of cast iron, so that

for the same price twice as much iron as steel, can be used in the bearings. Machine work done upon cast iron costs about the same as that for cast steel. The steel bearing is undoubtedly the strongest and most uniform casting; but even in cast iron bearings there is more metal than is really necessary.

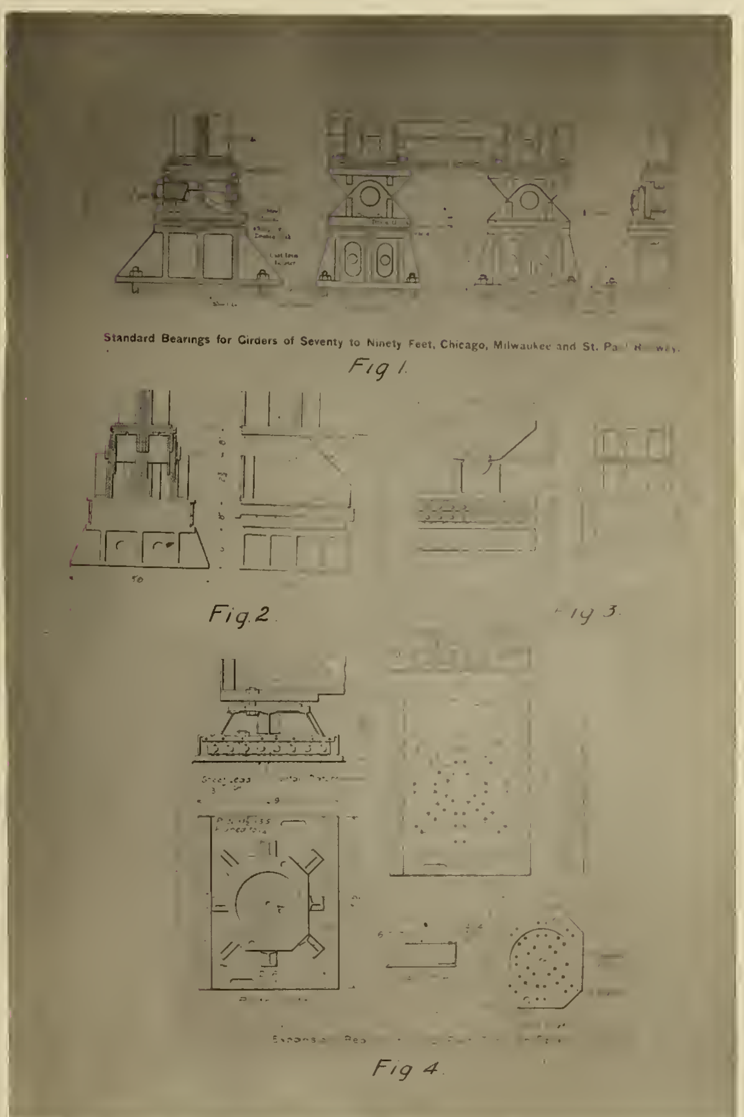
Bearings made of shapes and plates are quite frequently used. Pedestals of this type are often designed having a pin to transmit the load from the girder to the bearing, Fig. 1 and 2, Plate 6. This form of bearing uses the least amount of metal necessary, but it is doubtful whether it possesses the rigidity of the cast bearings.

The methods of allowing for the expansion of the structure are quite varied. The simplest expansion bearing consists of a smooth plate on the girder which can slide on another smooth plate that is fastened to the masonry. It is used on short girders.

The Chicago, Milwaukee and St. Paul Railroad uses a sliding bearing for girder bridges up to 90 feet in length. The form for 70-to 90-foot girders is shown in Fig.1, Plate 6. The shoe is cast steel and the bolster is cast iron. A phosphor-bronze disc forms the sliding surface. Fig. 2, represents a form of bearing made up of plates and angles. Expansion is taken up by segments, which are prevented from tipping beyond a certain angle by the bars on the sides. In some designs the segments rest upon T-rails placed paral-

PLATE 6.

Expansion Bearings



lel to the length of the bridge, as shown in Fig. 3.

The Canadian Pacific Railroad provides a spherical bearing for long span girders. Deflection, distortion, and expansion, are well provided for. A plate fastened to the girder is bored out to fit the convex surface of a cast-steel disc whose upper surface has been turned to the form of a spherical segment. The radius of the bored out portion is slightly larger than that of the segment. At the expansion end rollers protected from dirt by angles, are used. Fig. 4, Plate 6, shows the details for the bearing of a 100-foot through span.

Roller expansion-bearings are the most common type, but they have not proved satisfactory. In spite of all precaution dirt will accumulate in them and they will not roll. The segmental bearing is an attempt to provide more efficiently for expansion. Machine work on segmental bearings is a large item of cost. The Northern Pacific Railway specifies that the segmental surfaces and the surface upon which they rest, shall be very smooth. The bearing value per inch is the same as for circular cylindrical rollers, but the segments do not require so much space as the former.

It is common practice to make the bolster at the fixed end greater in depth than at the expansion end, by an amount equal to the depth of rollers or segments. The masonry at both ends can then be built to the same elevation. A marked difference in the height of pedestals is noticeable. The variations in some cases are due to the fact that one

engineer attemps to save masonry, while another endeavors to save metal.

VII.

FORM OF THE GIRDERS.

The usual form of girder is that of a simple rectangular beam. Such a structure is not pleasing to the eye. In order to vary the monotony of the design, the upper corners are sometimes rounded off. The cost of dies for this work is considerable, and it is desirable to keep the radius of the curve constant as far as possible for any one order for girders.

VIII.

THE SPACING OF THE GIRDERS.

The spacing of girders is a matter of much importance. One class of engineers favor a wide spacing, about 10 feet, for deck girders; another class require them spaced 6-1/2 and 7 feet upon centers. The former class usually varies the distance center to center according to the depth, while the latter use the same spacing for all spans. It is a question whether the stiffness and rigidity which is assured by wide spacing, compensates for the extra expense for larger ties which are required with it. J. A. L. Waddell, famed for designing staunch structures, places all long girders a distance apart, center to center,

equal to the nearest one half foot to one tenth of the span.

In the Technograph, published at the University of Illinois, for 1904-1905, an example of wide spacing is examined. The bridge noted is on the Duluth and Iron Range Railroad, over Little Sucker Creek, near Duluth. It consists of five spans each about 63 feet long, and two tower spans of 17-1/2 feet each. The bridge is of the deck girder type, and all girders are placed 10 feet upon centers. A comparison is made of the bridge as built, with a similar structure having girders 7 feet 6 inches on centers. It is shown that the actual bridge weighs 6-3/4% more than it would if the girders were placed 7 feet 6 inches on centers. There is also an increase of 84% on the decking. The example given shows the large increase in first cost due to wide spacing.

The advantages claimed for wide spacing are as follows:- (a) increased stability; (b) increased safety in case of derailment; (c) accessibility in case of accident, the wrecking crew having a much better opportunity to work around the train. The advantages of wide spacing are, in the opinion engineers, offset by the low first cost of a bridge with girders closer together. Ample rigidity is secured with girders 6-1/2, 7, or 7-1/2 feet on centers.

The spacing of girders for through bridges is determined by the usual clearance diagram, and the variations in different designs are not prominent enough to merit discussion.

IX.

WEIGHTS OF GIRDER SPANS.

There are very noticeable variations in the weights of standard girder bridges constructed by leading railroads. Plate 7 shows very clearly the variations. The curves representing Johnson's formulas($w = 12L + 150$ for deck girders, and $w = 12L + 500$ for through bridges), were derived by the method of least squares from actual weights of The American Bridge Company's medium-steel bridges. It is seen that the Northern Pacific Railway's medium-steel girder bridges approach very closely the weights given by these formulas. The weights of the Atchison, Topeka and Santa Fe Railway's deck girder bridges, class A, agree with the formula even more closely than do those of the Northern Pacific. The weights of the former's structures could not well be shown on the diagram. Their class B girder bridges consist of four shallow girders, and are of course much heavier than the class A spans. The Lehigh Valley Railroad's standard soft-steel bridges designed for Cooper's E50 loading, are very heavy. Their through bridges weigh about the same as class C of the Atchison, Topeka and Santa Fe Railway. Double-track deck-bridges consist of four girders and the weights are almost twice those given for single track structures. Plate 8 shows the difference between the weights of double-track deck-and through girder-bridges of the Lehigh Valley Railroad.

PLATE 7. DIAGRAM OF WEIGHTS
SINGLE TRACK GIRDER BRIDGES

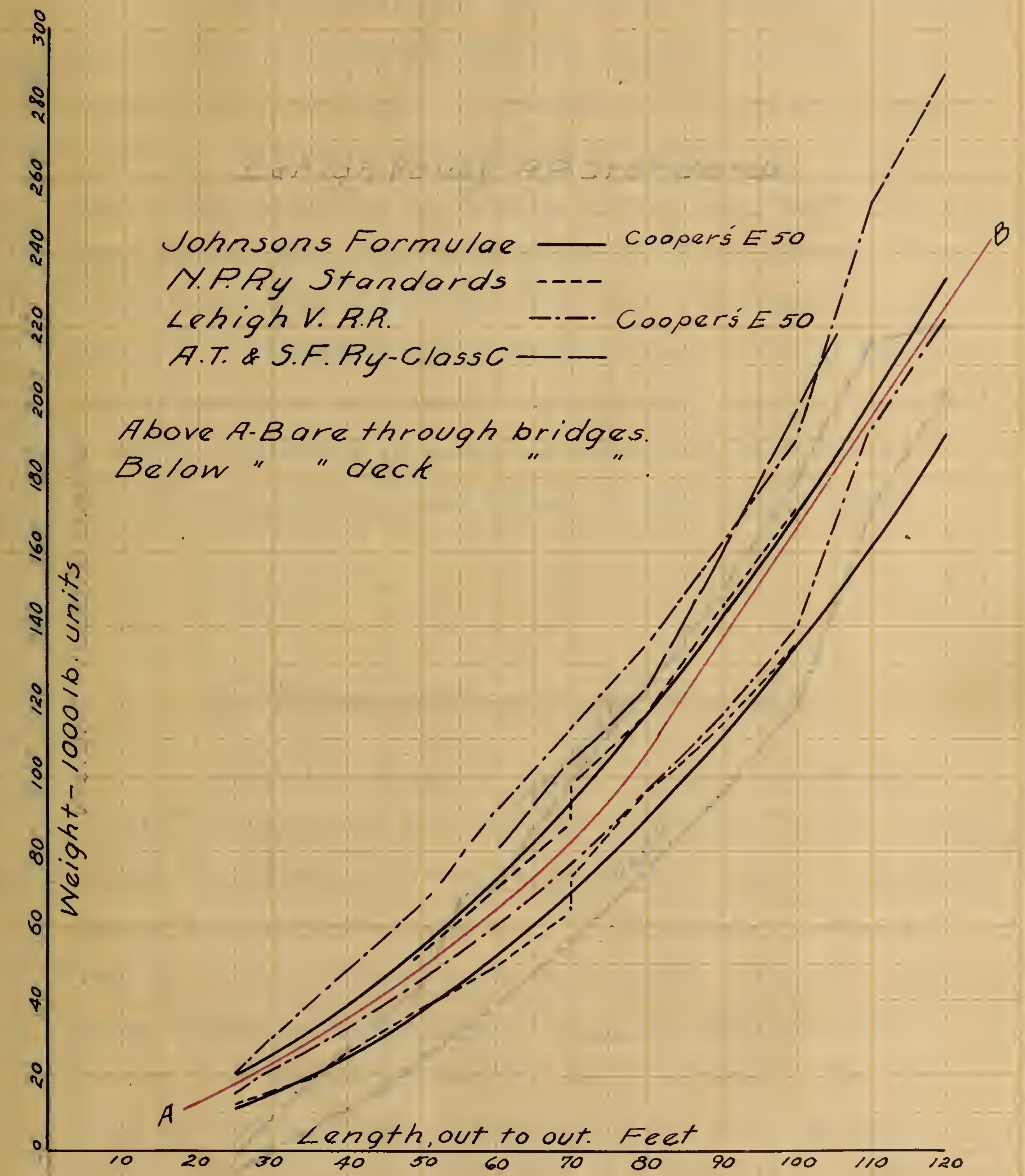


PLATE 7. DIAGRAM OF WEIGHTS
SINGLE TRACK GIRDER BRIDGES

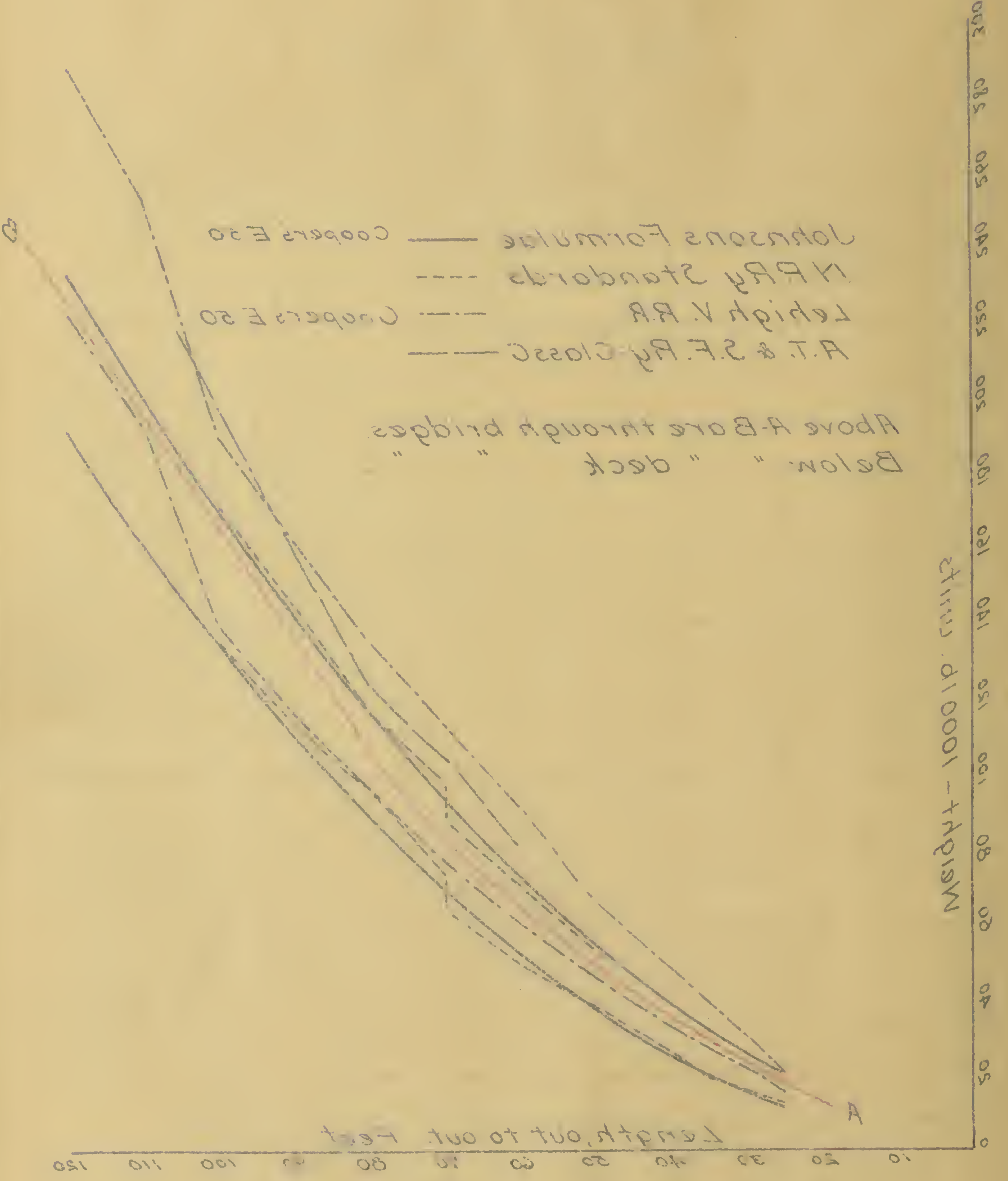


PLATE 8 DIAGRAM OF WEIGHTS

DOUBLE TRACK GIRDER BRIDGES

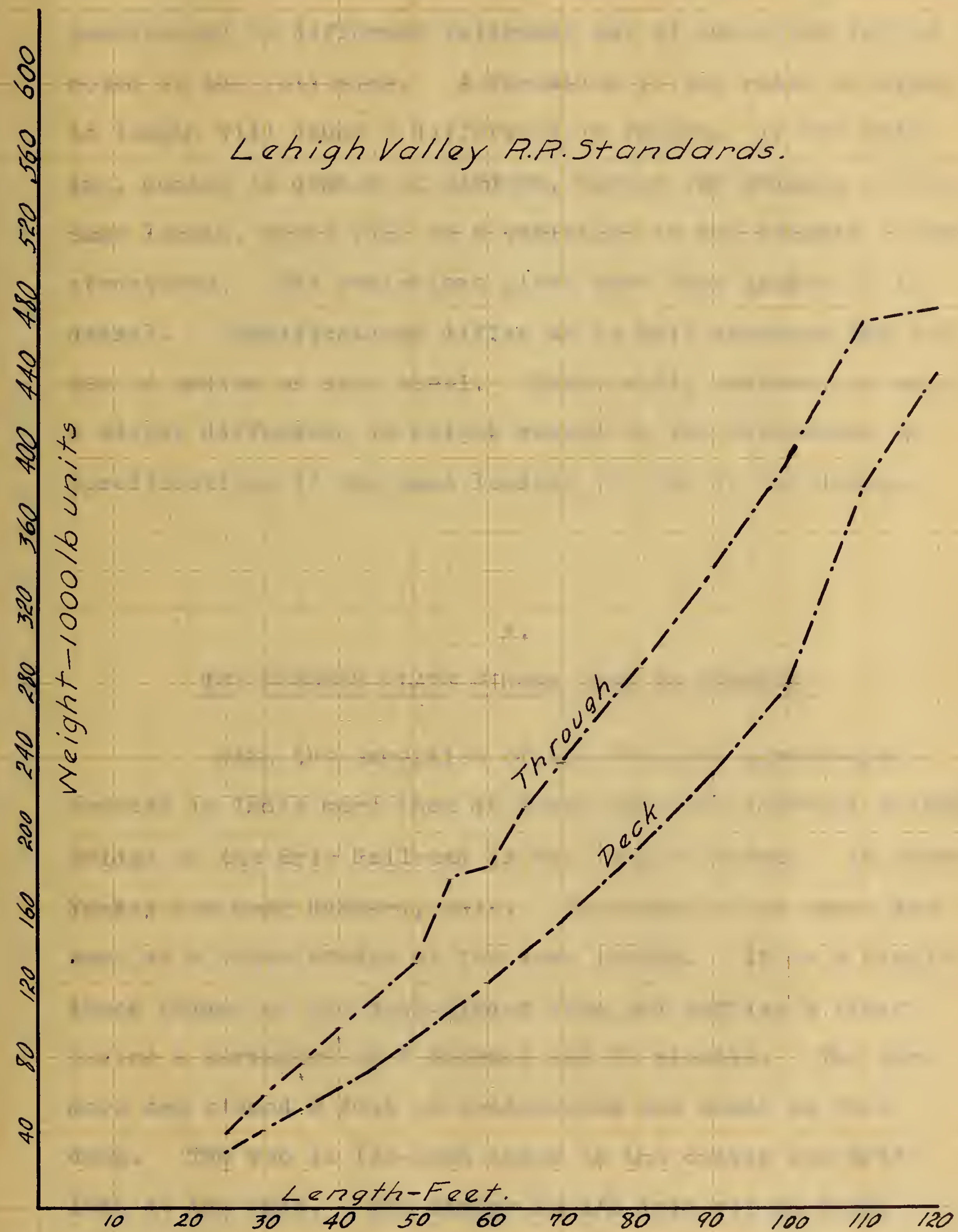


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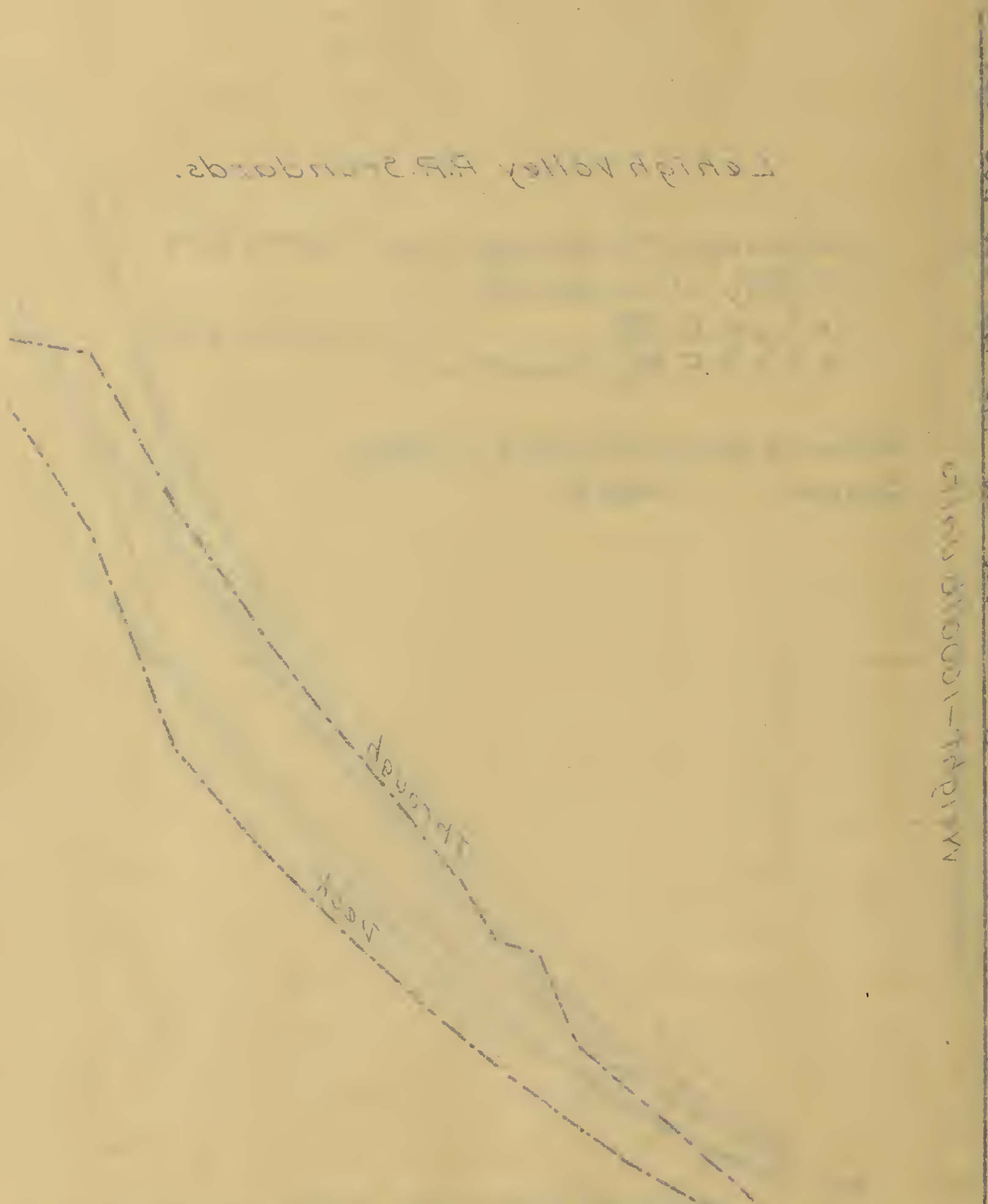
PLATE 8

DIAGRAM OF WEIGHTS
DOUBLE TRACK GIRDER BRIDGES

Lehigh Valley R.R. Standards.



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The variations in the weights of similar structures constructed by different railroads may be accounted for as noted in the following. A variation in the ratio of depth to length will cause a difference in weight. If the spacing, center to center of girders, varies for bridges of the same length, there will be a variation in the weights of the structures. The variations given have been spoken of in detail. Specifications differ as to unit stresses and the use of medium or soft steel. There will, however, be only a slight difference in weight caused by the difference in specifications if the same loading is used in the design.

X.

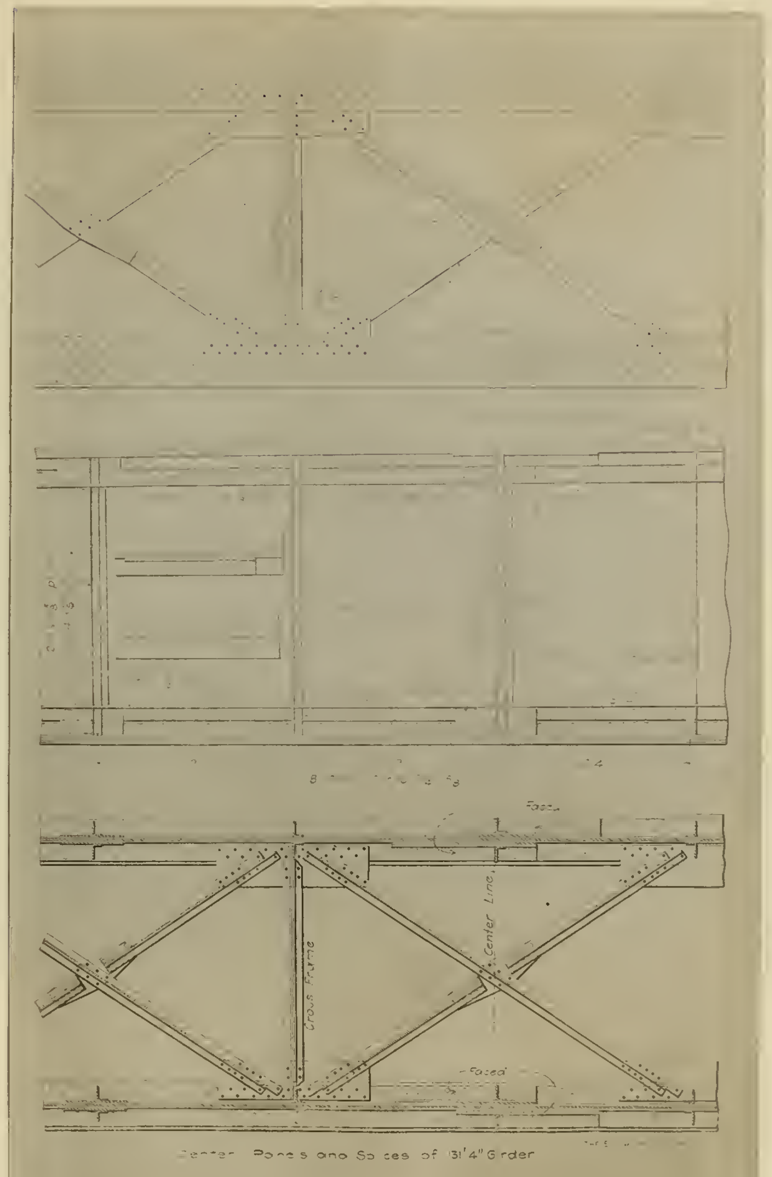
THE LONGEST PLATE GIRDER SPAN IN AMERICA.

With the exception of the 170-foot girder-span erected in India more than 40 years ago, the 131-foot 4-inch bridge of the Erie Railroad is the longest known. It spans Yankee Run near Hubbard, Ohio. In weight it is about the same as a truss bridge of the same length. It is a single-track bridge of the deck-girder type, and carries a track having a curvature of 3 degrees and 30 minutes. The girders are placed 9 feet on centers, and are about 10 feet deep. The web is 1/2-inch thick at the center and 9/16 inch at the ends. The camber is 1/8-inch per 10 feet.

The maximum flange section consists of four 6" x 8" x 7/8" angles and four 20" x 5/8" cover plates. The

PLATE 9.

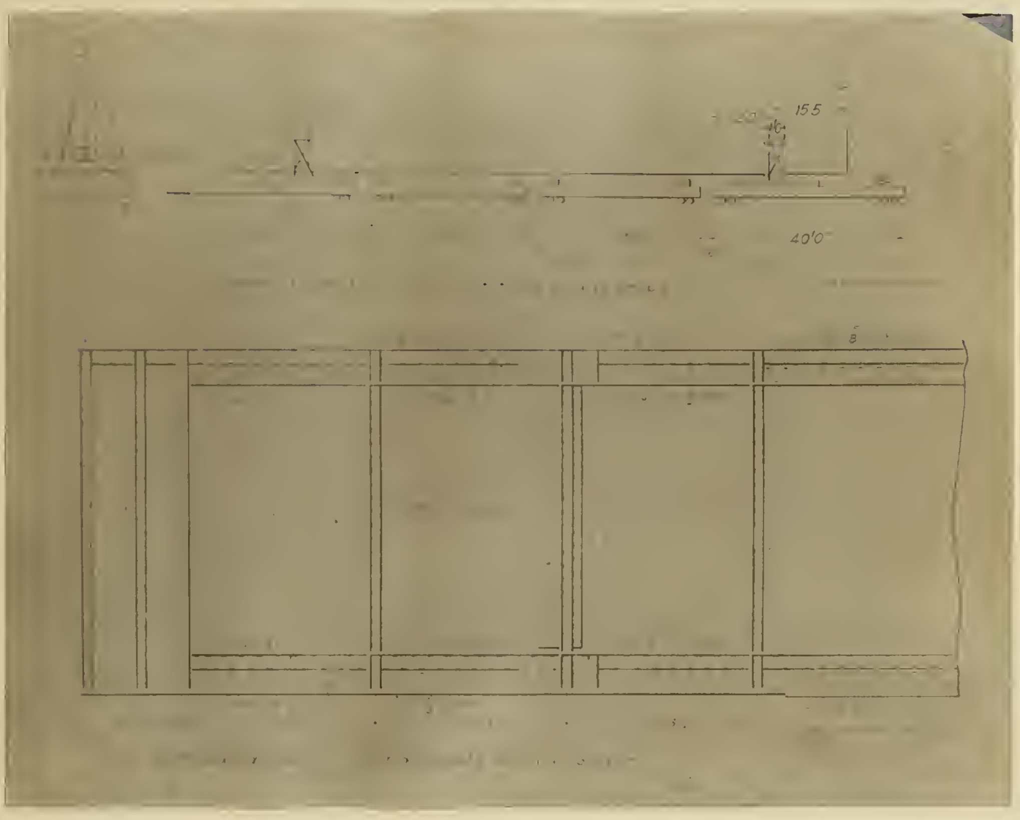
Details of Construction of The Longest Plate Girder Bridge in America



Details of Girder Construction and Bracing.

PLATE 10.

Details of Construction, and Method of Transporting The Longest Plate Girder Bridge in America.



first cover-plate extends to the end of the girder, and the lower angles are about 6 feet shorter. The shortest cover-plate is 64 feet 6 inches long. Splices in the flange angle are staggered, and placed symmetrical with respect to the center.

The cast steel pedestals used are placed 128 feet 4 inches apart on centers, and are fastened to the bottom flange with nine 1-inch tap-bolts. Those at the expansion end weigh 1,215 pounds each.

The girders were riveted up complete in the shop, and weigh about 113,700 pounds each. The main features of the girders and the method of transporting them are shown in Plate 9 and 10. Eight cars carried the two girders to the bridge site, and by taking special precautions it was possible to transport them around 6 and 8 degree curves, although at slow speeds.

For a complete description see the Engineering Record, Vol. 52, p. 324, Sept. 16, 1905.

XI.

SUMMARY.

Standard plate girder bridge designs of various railroads disagree on a great many points of importance. The main differences are as follows:

1. Ratio of depth of web plate to length of girder.
2. Maximum economical length of girder bridges.

3. The maximum spacing and size of stiffener angles.
4. Crimping of stiffener angles.
5. The proper form of top flange for deck girder bridges.
6. The ratio of the area of angles in the flange to the total flange area.
7. Per cent of the web area to be considered effective as flange area.
8. Shapes for lateral bracing.
9. Form of bearing at the expansion end.
10. Spacing of girders.
11. Weight.

A brief summary of the investigation upon the points noted will be given.

The limits of the ratios of depth to length, of a number of bridges examined are 1/7.1 as a maximum, and 1/13.7 as a minimum. It is conceded that the economical ratio is about 1/10. For short bridges the ratio is increased, and for bridges over 100 feet in length it is decreased. The ratios given below represent the best practice.

Lengths of Bridges feet.	Minimum ratio of depth to length.
25 to 50	1/9
50 to 75	1/10
75 to 100	1/11
100 to 125	1/12.5

It is believed that the above ratios will give economical structures.

The maximum economical length of girder bridge will vary for different railroads. The maximum curves will dictate the length of girder which can be transported economically. If each girder can not be riveted complete in the shop, the truss bridge will prove to be the preferable structure. The equipment for erecting girders must also be considered in this connection. The opinion is held that where maximum curvature and equipment are of minor importance, the economical length of single track girders is limited to those of such lengths that require web-plates which can be punched. There must, however, be a minimum number of web splices, in order not to unnecessarily increase the weight and cost of the structure.

The maximum spacing of stiffener angles is 5-feet in the best practice. It is agreed that end angles should have sufficient area to resist the end shear. The size of intermediate stiffeners may be given in the specifications, or their selection may be left as a matter of judgement. Practice is about evenly divided in regard to crimping and using fillers for intermediate stiffeners. End angles are nearly always fillered. The action of stiffeners is not well understood, and is a matter which requires further investigation.

Top flanges, for deck bridges, composed of angles only, are used on a few roads. They are not generally accepted because of the well formed belief that the flange

angles require protection from weathering and direct action of the wheel loads. In order to give a maximum stiffness at least one half of the total flange area must be in angles. On short girders the area of angles is often more than one half of the total flange area. For long girders the reverse is true. On the girders investigated the area of flange angles varied from 0.69 of the area of plates plus angles for a 26-foot bridge, to 0.24 for a 131-foot bridge. It is common practice to consider 1/8 of the area of the web plate as effective flange area. When the web requires splicing some designs require that no portion of it shall be regarded as effective flange area.

For lateral bracing, angles are almost universally used. They are easily handled and will not sag. Plates are cheaper and easy to handle, but will sag unless supported. Rods are never used in modern designs because of the large decrease in strength caused by weathering, and a tendency to vibrate and rattle.

There are many who advocate the use of the segmental bearing. Its cost is more than that of the roller bearing, but if properly designed to prevent tipping too far, its efficiency is such that the extra cost appears to be justified. Roller bearings are liable to become clogged. Cast iron shoes are not as costly as those of cast steel, but give ample strength.

Some engineers space the girders for deck girders 10 or 10 1/2 feet upon centers, while others space them 6 1/2, 7, or 7 1/2 feet, center to center. The

arguments in favor of wide spacing are:- (1) increased rigidity of structure, (2) increased safety in case of derailment, (3) more room for the wrecking crew to work, in case of a wreck upon the bridge. To offset the above- (1) the cost of the metal is decreased appreciably by narrow spacing, (2) the cost of the floor system is decreased, (3) and ample rigidity is secured with the spacing of about 7 feet.

The weights of girders vary with the ratio of depth to length, spacing, and specifications concerning unit stresses and kind of metal to be used. J. B. Johnson's formulas give weights very close to actual weights of economical designs.

XII.

CONCLUSION.

The plate girder bridge is a comparatively new design and there are many points in regard to it upon which engineers do not agree. However, this type of structure has given general satisfaction because of its safety, rigidity, and ease of erection and construction.

The fact that the details may be varied considerably and still leave the structure safe, durable, and reliable, is a great point in its favor; indeed we might say that it has elasticity of design. The design is simple in comparison with that of other bridges. For spans of from 25 feet to 125 feet, the girder bridge is a most desirable structure.